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VERIFICATION OF PERFORMANCE PREDICTION MODELS AND DEVELOPMENT OF DATA BASE PHASE II ARIZONA PAVEMENT MANAGEMENT SYSTEM

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PAVEMENT MANAGEMENT SYSTEM
FOR
ARIZONA PHASE II

VERIFICATION OF PERFORMANCE PREDICTION MODELS
AND
DEVELOPMENT OF DATA BASE

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16. Abstract

A pavement management system (PMS) has been defined as "the systematic development of information and procedures in optimizing the design and maintenance of pavements" (1). The purpose of this research was to verify and adjust models (equations) developed in phase I, II and III of the PMS. This verification process involved testing models against real data and determining the correlation. Appropriate adjustments were made to enhance the final predictions. Results of this work indicate that the prediction models can reasonably predict the future ride and cracking condition for newly constructed, in-service and overlaid asphaltic concrete pavements, as well as, plain concrete pavements.

The second purpose of this project was to develop a PMS data base. Such a data base was developed through a cooperative effort between Research Section, Materials Services and Information System Groups. The data base contains over 250,000 records which are stored in an information management system (IMS) file. Data is stored hierarchially which facilitates the retrieval of data via a remote terminal. Computer programs which allow various users (Designers, Maintenance Engineers, District staff, Researchers, Planners and others) to retrieve data in less than one minute have been implemented and have been in use for six months within the department.

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PAVEMENT MANAGEMENT SYSTEM (PMS)

ADOT is currently engaged in implementing PMS. At present, no state has been able to make a complete PMS operational. ADOT is very close to creating a fully functional operational PMS; however, one basic element of such a system is not currently present and that is a PMS Operations Group.

Before defining the exact size or structure of a PMS Operations Group, a brief history of how ADOT has managed to come this far is in order. In June 1978, ADOT, through the State Engineer, made a commitment to develop a PMS and make it functional. A group of knowledgeable co-principals was charged with literally creating the PMS within ADOT. To help this group, six temporary positions were furnished. Over the months, additional help was obtained from Information Systems. Virtually all of the work effort up until now has been done on a cooperative level. In all, 21 positions have dedicated more than 50 percent of their time to completing this project as shown below.

Positions Working on PMS

Research Section	1	
Materials Services	10	(Includes Inventory)
Information Systems	4	
Temporary	6	
Total	21	

Since different lines of authority have been involved, some time has naturally been spent in working out priorities and solving personnel problems. Two of the temporary positions soon will be terminated. In addition, work by the consultant will be complete by January 1980. Findings by both the consultant and the ADOT staff indicate that the inventory work effort will change in the future. Greater emphasis should be placed on obtaining the most current ride, percent cracking and skid number. Deflection measurements most likely would be performed as a design test on an as-needed basis. Considering the above, an attempt has been made at visualizing the PMS Group of the future (1980).

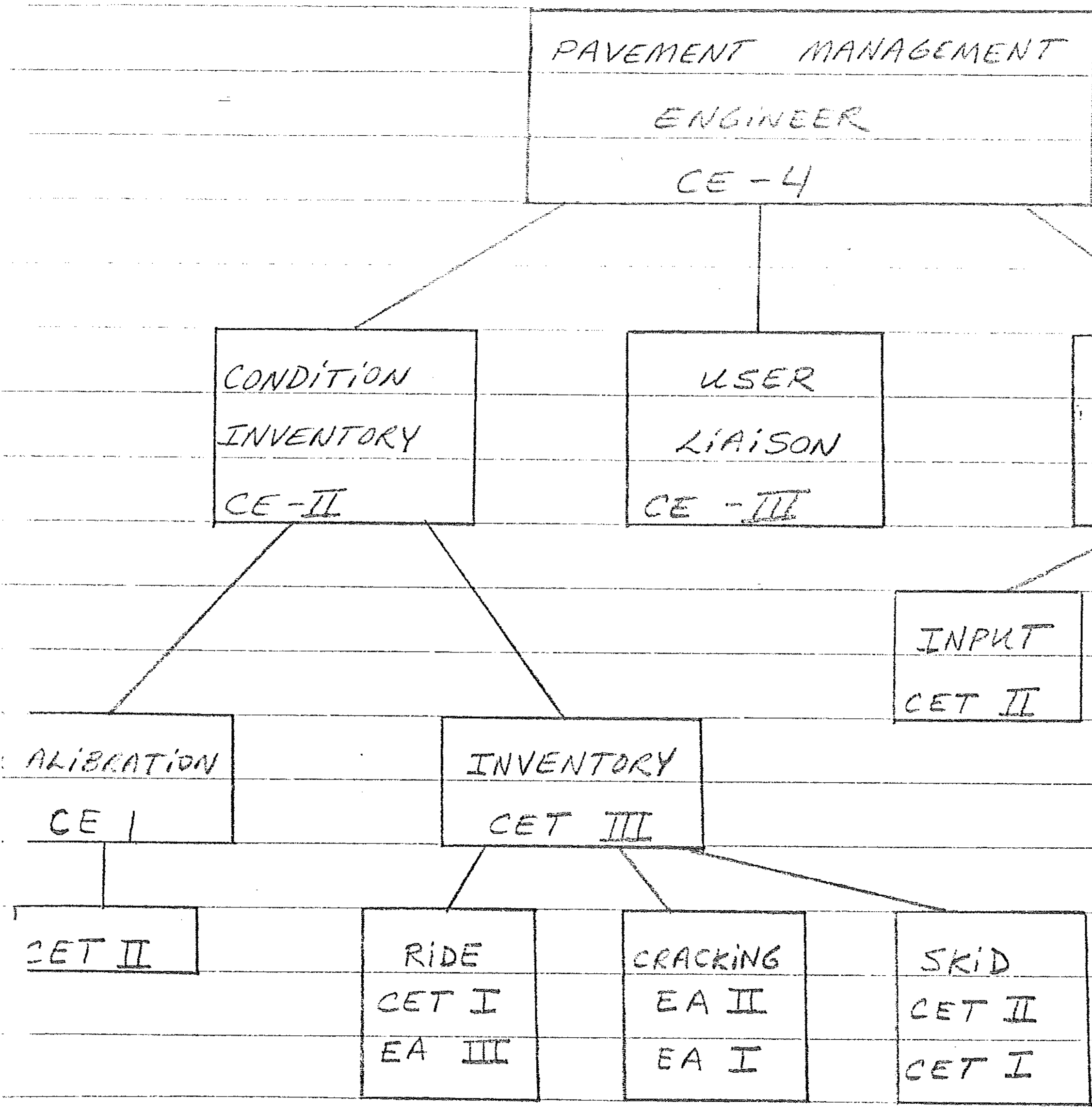
Attached is an organizational chart representing a future 1980 PMS Group. Since PMS would furnish both information to a variety of users (Traffic, Planning, Operations, Districts and Research) and create a future preservation plan of action, it would be advantageous that it report to the Chief Deputy or State Engineer. The top position could be either a CE-4 or 5. Supporting the top position would be two functional areas denoted as condition inventory and analysis. Both areas would be headed up by a CE-2 or equivalent management position in the case of analysis. The role of the condition inventory arm would be to collect inventory ride, cracking and skid data in the field. This area would not perform deflection test as this would be a design function and could be performed by a design crew or District on an as-needed basis. Those persons working in the inventory area would be responsible for equipment upkeep, standardization and maintenance. In addition, until that time when all tests are automated, they would perform manual coding.

The analysis area would consist of those office personnel needed to input and output all data for the PMS data base. Input data would consist of condition data, highway history (new construction, overlay, seal coats, etc.) and construction data. In addition, data from other existing files such as PECOS, Traffic ADT and ADL would be updated and input. In the future, other files might be created such as construction costs, geometrics, etc. Output work would involve providing most current data to users (updating), future preservation action plan, reports to Highways Division management, annual summary condition report, priority planning report, three R study, interstate needs study and other special studies aimed at improving design, construction and maintenance. The liaison feedback position would be responsible for answering user questions and needs. Questions from users such as Districts, design, etc., would be fed back to the inventory or analysis area to see where errors were made or to make improvements. Likewise, this position would be responsible for explaining new innovations in the system to user groups as well as passing results of studies back to each group.

In all, 17 positions, including two typists, would be needed.

FUTURE

PAVEMENT MANAGEMENT GROUP



ABSTRACT

A pavement management system (PMS) has been defined as "the systematic development of information and procedures in optimizing the design and maintenance of pavements" (1). The purpose of this research was to verify and adjust models (equations) developed in phase I, II and III of the PMS. This verification process involved testing models against real data and determining the correlation. Appropriate adjustments were made to enhance the final predictions. Results of this work indicate that the prediction models can reasonably predict the future ride and cracking condition for newly constructed, in-service and overlaid asphaltic concrete pavements, as well as, plain concrete pavements.

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INTRODUCTION

A pavement management system (PMS) has been defined as "the systematic development of information and procedures necessary in optimizing the design and maintenance of pavements" (1). As ADOT's highway network has grown and reached completion, the concern of highway engineers and managers has shifted from new construction to preserving the existing highway network. At present, ADOT has over 6,000 miles of highways within its system. The cumulative cost to construct the present system was about $2.1 billion. To replace the existing highways at today's dollars would amount to $4.0 billion; however, to overlay the entire system would cost about $500 million. The idea behind PMS is that it is possible through a systematic management methodology to preserve the condition of ADOT's highways at or above an acceptable level at a reduced cost.

To implement PMS within ADOT has involved three phases:

Phase I	Develop program to optimize the design of new construction and major maintenance completed by Woodward-Clyde Consultants in 1976 (1).
Phase II	A. Verify prediction models with actual data and create computerized data base.
	B. Develop a functional PMS within ADOT. To be accomplished by ADOT staff by March 1981.
Phase III	Develop a network optimization system. To be developed by Woodward-Clyde Consultants and tested by ADOT staff.

Phase II and III projects represent a joint effort between ADOT and Woodward-Clyde Consultants. Information, highway condition data and general overall direction of both projects was managed by a series of meetings between principal investigators. In addition to this ADOT created a management steering committee composed of the following positions.:

Chief Deputy Engineer - Chairman
Assistant State Engineer Traffic
Priority Program Manager
Maintenance Engineer
Materials Engineer
Information Systems Project Manager

This committee addressed important operational problems and recommended appropriate actions to be taken to the State Engineer.

The purpose of this part of the Phase II project was to verify and adjust existing models and develop a suitable data base for the use of the Phase III program as well as design, maintenance and management.

Model Verification

In Phase I Woodward-Clyde developed pavement performance prediction models by using the Bayesian method (1). Models were created by interviewing knowledgeable highway engineers about their expectations of future pavement performance in terms of several variables. From these values mathematical models (equations) were developed and are shown below.

1976 MODELS

New and In-Service Construction

$$LN\ CRI = 0.8815\ LN\ RGN + 0.6965\ LN\ DEFL + 0.1901\ LN\ TRAF + 0.4217\ LN\ AGE + 1.6638$$

Where
CRI = Change in Roughness Index in Two Years
RGN = Environmental Region
- 1 = 0 to 5000 feet elevation
- 2 = Greater than 5000 feet
- 3 = Greater than 5000 feet with swelling clay foundation

DEFL = Equivalent Benkelman Beam (BB) Deflection Obtained From Correlations with Dynaflect Deflection (.001 inch Dynaflect = .0224 inch BB)

TRAF = Average Annual Equivalent 18 Kip (8 kn) Single Axle Loads Estimated for the Specific Roadway

AGE = Age of Pavement In Years

For an overlay plus an asphalt concrete friction course without an asphalt-rubber inner-layer or heater-scarification

$$LN\ CRI = 0.8744 \ln RGN + 0.3281 \ln DEFL + 0.0718 \ln TRAF - 0.0375 \ln THIK + 0.4618 \ln AGE + 1.2736$$

Where
CRI, RGN, DEFL, TRAF, and AGE are the same as used in equation (1) and

THIK = thickness of the overlay

For new construction or overlays

$$LN\ CSN = 0.2940 \ln RGN - 1.0046\ AGE + 0.6949 \ln AGT + 0.0594 \ln TRAF + 1.9420$$

Where
CSN = annual change in skid number
RGN, AGE, and TRAF are the same as used in equation (1) and AGT = type of aggregate; 1 for basalt, 2 for gravel, and 3 for limestone.

Ride index immediately after overlay was related to the ride index before and thickness.

$$LN(RI_a) = 1.628 + 0.309\,LN(RI_b) - 0.237\,LN(THIK)$$

Where
RI_a = Ride Index After Overlay
RI_b = Ride Index Before Overlay
THIK= Thickness of Overlay

When an overlay was built with asphalt rubber or heater scarification a correction factor called CRH was used to reduce the amount of change in ride per year. Since completion of the 1976 project a CRH of .7 was used for heater scarification and .6 for asphalt rubber.

Roughness index was defined as the Mays meter roughness, however, in 1976 this value was interpreted differently than at present. To convert the 1976 value to the correct value it must be multiplied by 6.4.

The above represented ADOT and Woodward-Clyde's best approximation of future ride and skid number. During two and one half years of using these equations it became obvious that a percent cracking prediction model was needed as well as an improved ride model based on real data. The skid number prediction model, although technically correct, was always predicting no future problem due to aggregate abrasion, nevertheless serious low skid numbers did occur evidently for other reasons. Generally these reasons were of an uncontroll-able nature at the construction site or maintenance activity. With these historical experiences in mind it was decided in this project to develop prediction models for both roughness and percent cracking. Skid numbers would not be predicted, but rather monitored closely to determine those miles of highway in need of fix up. It is hoped that historical construction and maintenance data accumulated as part of this project will in the future be able to identify and correct the reasons for low skid number.

Factorial Design

Since results of the Phase II work would be incorporated into the Phase III, discussions were held to set guidelines for the new prediction models. These guidelines included the following:

1. Models (equations) should be able to predict next years ride and percent cracking very accurately. This was necessary because those highways to be overlayed next year will be in next year condition at the time of overlay, also annual monitoring of condition would insure that next years values would be known.
2. Models should be able to predict reasonably well for a four to five year time frame. This would fit into the five year plan which ADOT must compile and present to the ADOT commission and Governor for approval each year.
3. Models should contain no more than five independent variables; preferrably less. In this way the size of the network problem could be kept within reason.
4. Models should predict in one year increments.

With these guidelines, an incomplete factorial experiment was designed by Woodward-Clyde and is shown on Figure 1. Originally only projects built since 1969 were going to be incorporated into the project. 1969 represented a year when a new set of specifications were published, also the design of

FIGURE 1

1/3 FACTORIAL DESIGN: FILL THE CELLS WHICH ARE MARKED.
(Requires a total of 27 units.)

Ride Index	Deflection	Environment 1			Environment 2			Environment 3		
	Age in Years	3 to 7	8 to 12	13 to 17	3 to 7	8 to 12	13 to 17	3 to 7	8 to 12	13 to 17
H	H		X		X					X
H	M	X					X		X	
H	L			X		X		X		
M	H	X					√		X	
M	M			X		√		X		
M	L		X		X					X
L	H			√		X		X		
L	M		√		X					√
L	L	X					X		√	

X = Main Experiment √ = Replicates

asphaltic concrete (AC) changed. It was not possible to fill more than half of the cells, the sample was changed to increase the time frame from 1963 to the present. 1963 was selected because it represented that time when the AASHTO Interim Guidelines (2) were put into practice. The selection process was widened to include any mile of highway built since 1963 and a mile could represent more than one cell, as its condition changed with time. Unfortunately the cell design was unsatisfactory in solving the problem due to the use of ride index and deflection as factors that were constrained or bracheted into region. A substitute factorial scheme was devised. In this new scheme region and time were divided into three levels as shown below.

Regional factor (AASHTO)	0 - 1.6	Desert
	1.7 - 3.0	Transition
	3.1 - 5.0	Mountains
Age of AC Pavement	0 - 5.0	Years
	5.1 -10.0	Years
	10.1 -15.0	Years

This produced nine combinations. For each combination 15 different miles were randomly selected, giving a total of 135 miles of new construction and 135 miles of overlays. Thus each sample represented about 2.3 percent of the miles in the system. Woodward-Clyde advised that this was a more than adequate sample size. In addition those miles where all data were present were also included. That is if roughness, cracking and deflection data were present for years 1973, 1975 and 1979, all of these years of data were included under the same milepost. Appendix A gives a description of the data as well as all the data used to generate future correlations. Basically this data included the following.

- Route number
- Direction
- Milepost
- Cell number
- Record year - year condition tests performed
- Regional factor - AASHTO regional factor, derived from elevation, rainfall and climate zone.
  .1 of a point for each 1000 feet of elevation,
  .1 of a point for each inch of average annual rainfall and .1 for climate zones as shown on Figure 2.

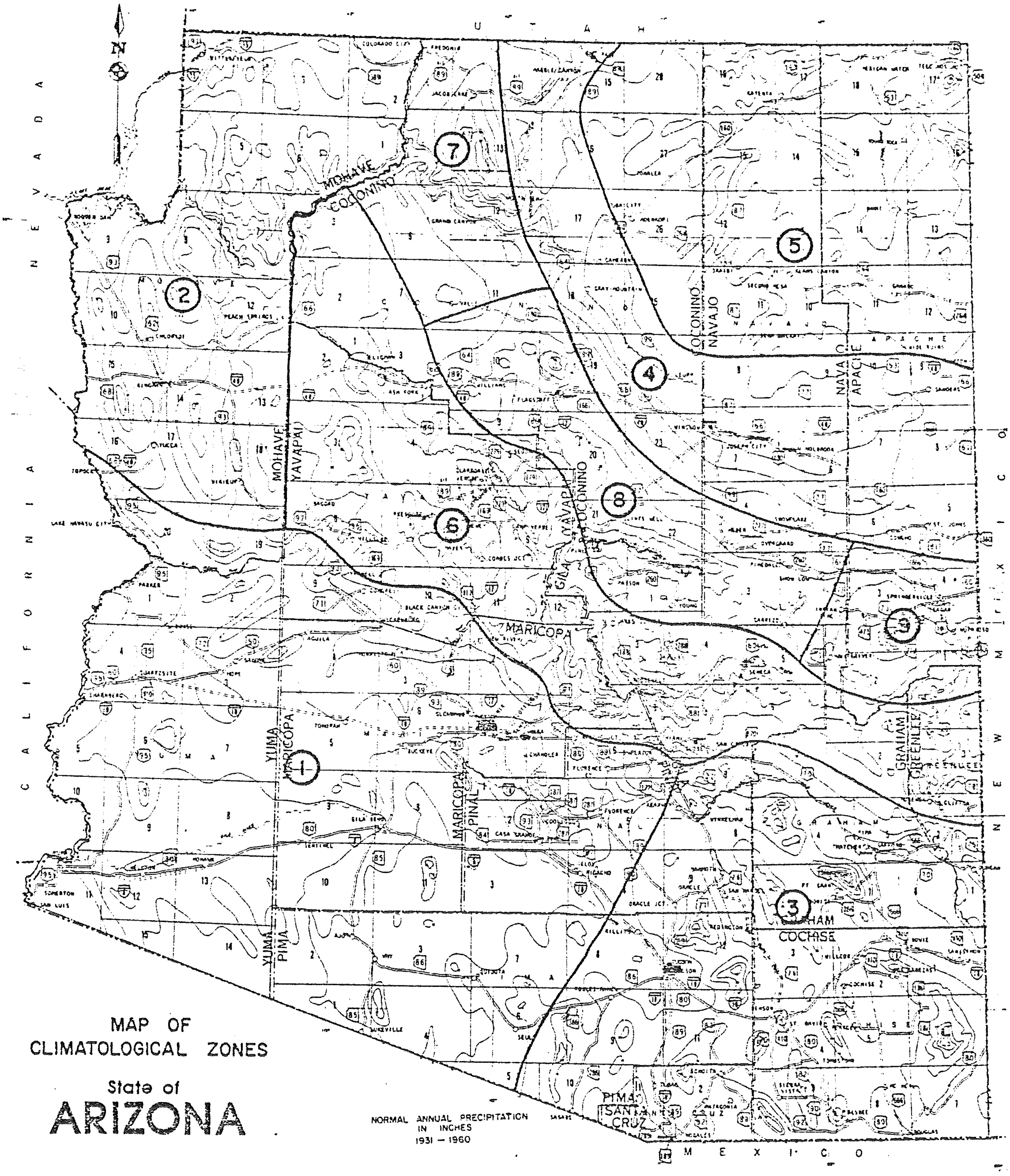
- Thickness of original AC surfacing in inches
- Thickness of AC overlay in inches
- Number of single axle equivalent 18 kip (80 kn) traffic in year of record

- % cracking in year of record
- % cracking one year after year of record

- Mays meter inches of roughness in year of record
- Mays meter inches of roughness one year after year of record

- Dynaflect deflections in milliinches (1.0 is equal to .001 inches of deflection) for all five geophones. All deflections were

FIGURE 2



N
UTAH
NEVADA
CALIFORNIA
NEW MEXICO
MEXICO
MOHAVE
COCONINO
YAVAPAI
GILA
MARICOPA
YUMA
PIMA
PINAL
SANTA CRUZ
COCHISE
GRAHAM
GREENLEE
NAVAJO
APACHE
1
2
3
4
5
6
7
8
9
MAP OF
CLIMATOLOGICAL ZONES
State of
ARIZONA
NORMAL ANNUAL PRECIPITATION
IN INCHES
1931 – 1960

temperature corrected according to the Asphalt Institute method (3).

- Age of pavement according to the year of record. If year of record 1976 and age 8 years, then pavement was built in 1968.

Consideration was given to other variables such as soil support, unbound base thickness, density, moisture, grading, asphalt content and asphalt type, however, either values for these variables were missing or too much uncertainty surrounded their determination. That is new construction or overlay asphalt content and type might have been found, a question would arise as to how applicable they would be to pavements 10 years old which have been flushed or seal coated, thus they were not investigated.

A number of regression runs were made to determine correlation to either the roughness or percent cracking directly from the other variables. New variables were created which included spreadability index, surface curvative index, base curvative index to name a few. Direct correlation of all variables to either the magnitude of roughness or percent cracking gave very poor results. An approach similar to the 1976 equation was attempted, which included the use of the change in roughness (ΔR) and change in percent cracking (Δ %C) per year. This approach developed equations which represent the new predictive equations based on real data.

New Models (Equations)

The models developed represent prediction of future roughness and percent cracking conditions based upon past experience. These models are intended to be used in conjuction with annual pavement condition surveys. These models are not design equations because they do not give any insight into what caused the new future distressed condition. Rather they represent a what system of examination and prediction. That is given what happened they predict what will happen. Design equations are why systems which represent why particular failures occur and develop design strategies to prevent or delay such occurrence. The following predictive models were developed and represent ADOT's future predictive models

New and In-Service Construction

Percent Cracking

$$\Delta \%C_n = 0.55(\Delta \%C_p) + 0.031(\%C_p * \%C) + 0.01(R_g)^2 + 0.05(R_g * \%C) - 0.0059(\%C)^2 + 0.186$$

$R^2 = 0.70$; Standard Error = 0.64, F Value = 84

Where

$\Delta\%C_n$ = Change in Amount of Cracking During Next Year
$\Delta\%C_p$ = Change in Amount of Cracking During Previous Year
%C = Present Amount of Cracking
R_g = Regional Factor

As an Example, Given:

1976 Percent Cracking = 10
1977 Percent Cracking = 15

Change in Percent Cracking = 5
Regional Factor = 2.0

Find the 1978, 1979 and 1980 Percent Cracking

Year	% Cracking	Change In % Cracking
1976	10	5
1977	15	5
1978	20	7
1979	27	7
1980	34	

Roughness

$$R_n = 0.138(R) + 2.65(Rg)^2 - 0.047(Rg*R) - 0.125$$

$R^2 = 0.54$, Standard Error = 10.4, F-Value = 38

Where

R_n = Change in Roughness During Next Year
R = Present Roughness
R_g = Regional Factor

Example, Given:

1976 Roughness = 100 inches/mile
Regional Factor= 2.0
Find the 1977, 1978, 1979 Roughness

Year	Roughness	Change in Roughness
1976	100	15
1977	115	15
1978	130	17
1979	147	

Naturally each year new roughness and cracking values would be measured in the field thus the starting value or seed value would change to reflect the real world value.

Percent Cracking — Overlay

$$\Delta\%C_n = 0.51 + 0.069(\%C) + 0.52(\Delta\%C_p) - 0.0034(D_L)^2 - 0.003(\%C)^2 + 0.068(\Delta\%C_p)^2$$

$R^2 = 0.68$, Standard Error = 0.71

Where

All symbols mean the same as before except one new term has been added.

D_L = Index to first year of cracking. Factor which represents the relative amount that each overlay and overlay plus treatment delays the first crack. Appendix B gives the index values for all treatments.

It should be noted that immediately after an overlay, both %C and Δ%Cp are set equal to zero to predict the change in cracking in one year. The term D_L accounts for benefit derived by using various treatments to prevent reflective cracking and is similar to the use of CRH in the 1976 Woodward-Clyde model.

EXAMPLE:
Given 1976 existing highway with

Regional Factor	= 2.0
Traffic	= 4000 ADT
Present Cracking	= 20 Percent
Change in Cracking in last year	= 3

2.5 inch AC overlay would have an index to first crack of 6.5.

Find Percent Cracking In Years 1977 to 1984

Overlay One Year	Year	Percent Cracking	Change In Percent Cracking
	1976	20	3
Overlay	1976	0	0
(1)	1977	0	0
(2)	1978	1	1
(3)	1979	2	1
(4)	1980	3	1
(5)	1981	4	1
(6)	1982	5	1
(7)	1983	6	1
(8)	1984	8	2
(9)	1985	9	1

Roughness

For an overlay the roughness change was found to be related to the roughness before overlay.

$$R_N = 65.29 - .78(R_B) - 7.76(TH)$$

$R^2 = .9379$

Where

R_N = Change in roughness one year following an overlay in inches/mile

TH = Thickness of overlay in inches

R_B = Roughness before overlay

Note: If calculated roughness after overlay less than 50, roughness set to 50.

After overlay at which time the in-service equation is used to perform future calculations.

EXAMPLE:

Given a 1976 pavement with the following conditions

Roughness = 200 inches/mile
Regional Factor = 2.0
Overlay thickness of 2.5 inches of AC

Find Roughness for 1977 Through 1985

	Year	Roughness	Change In Roughness	Overlay
	1976	200		Roughness Model
One Year	1976	Overlay		
After	1977	90	110	
(2)	1978	104	14	In-Service Roughness Model
(3)	1979	120	16	
(4)	1980	135	15	
(5)	1981	152	17	
(6)	1982	169	17	
(7)	1983	187	18	
(8)	1984	205	18	
(9)	1985	225	20	

For both roughness and percent cracking the actual amount one year after construction will be monitored. In order to test the accuracy of future predictions a verification process were undertaken.

Verification

Twenty nine miles of new construction or in-service pavements as well twenty four miles of overlays were randomly selected from the ADOT file.

Appendix C gives each mile, as well as pertinent data about each mile. A verification test was conducted by comparing expected future predicted roughness and percent cracking to actual measurements. In addition the predicted 1976 roughness derived from Woodward-Clydes original equation was also calculated. Since many projects were designed using the AASHTO equation the predicted present serviceability values were also calculated.

To test the equations it was necessary to conduct two separate calculations.

1). Given some starting roughness value (50 inches per mile) and cracking value (0 percent cracking) representative of the pavement immediately after new construction or overlay calculate the expected future ride and cracking and compare to the actual value.

Examples:
Case 1 - Given a mile of highway built in 1970 assume the new ride equals 50 inches per mile and 0 percent cracking.

Year	Actual Ride	Calculated Ride	Actual % Cracking	Calculated % Cracking
1970	42	50*	0	0*
1971	57	55	0	1
1972	63	60	1	2
1973	70	65	1	3

* 50 and 0 assumed.

Case 2 - Given some existing ride or % cracking condition calculate ride or % cracking in a future year.

Example: Given a mile of highway find the actual measured ride and % cracking for a given year. Use this measured value to calculate the ride or % cracking in a future year.

Ride			Calculated	Ride
Year	Actual Ride	Given 1972	Given 1973	Given 1974
1972	69			
1973	75	77		
1974	86	90	87	
1975	103	110	105	100

% Cracking

Year	Actual % Cracking		Calculated	% Cracking
1973	5	Given 1973		
1974	7	8	Given 1974	
1975	9	12	10	Given 1975
1976	15	16	14	13

To interpret results of the above analysis regressions between the actual and calculate ride and percent cracking were performed. This is quite straightforward for case 1, however, for case 2 actual and calculated values were grouped by year. Thus all one year predictions were grouped together. likewise all two year, three year and so forth.

To thoroughly examine the worth of the prediction equations, similar analysis were performed with the old SOMSAC equations and the present servicability (PSI) equation. Appendix D gives a summary of information for each site by site number, Information includes:

- Site location - Route, milepost, direction
- 18 kip single axle Equivalents in 1978
- Structual numbers
- Soil support
- Regional factor
- Beginning PSI
- Traffic Growth factor
- Year built
- % cracking
- Rut depth By year
- Ride
- PSI

To derive structual number for new construction AC was given a coefficient of .40 and base .12 per inch of thickness. For overlays the new AC was given a coefficient of .40 per inch and the old AC a coefficient of .20 or half the new value per inch. Existing PSI values were derived by using correlations determined in an earlier report (4). These correlations relates ride roughness to slope variance and Arizona percent cracking to ADOT class 2 and 3 cracking. Figures 3 and 4 show these relationships. Calculations from the raw data represented about 180 pages of values, therefore, summaries of the calculations are reprinted here Appendix D. Predicted versus actual roughness and percent cracking figures with correlations, standard errors and coefficient of variation are shown in Appendix E by site number.

FIGURE 3

MAYS RIDE METER ROUGHNESS VS. SLOPE VARIANCE

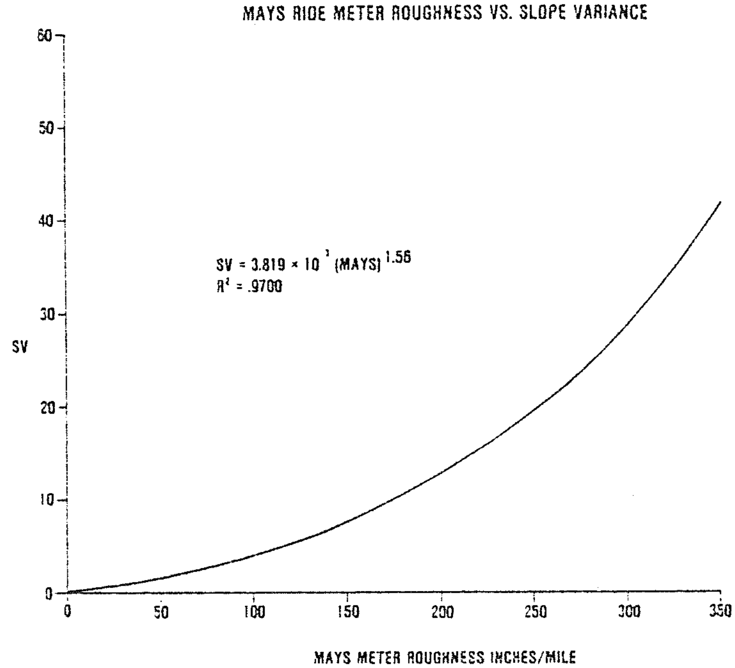
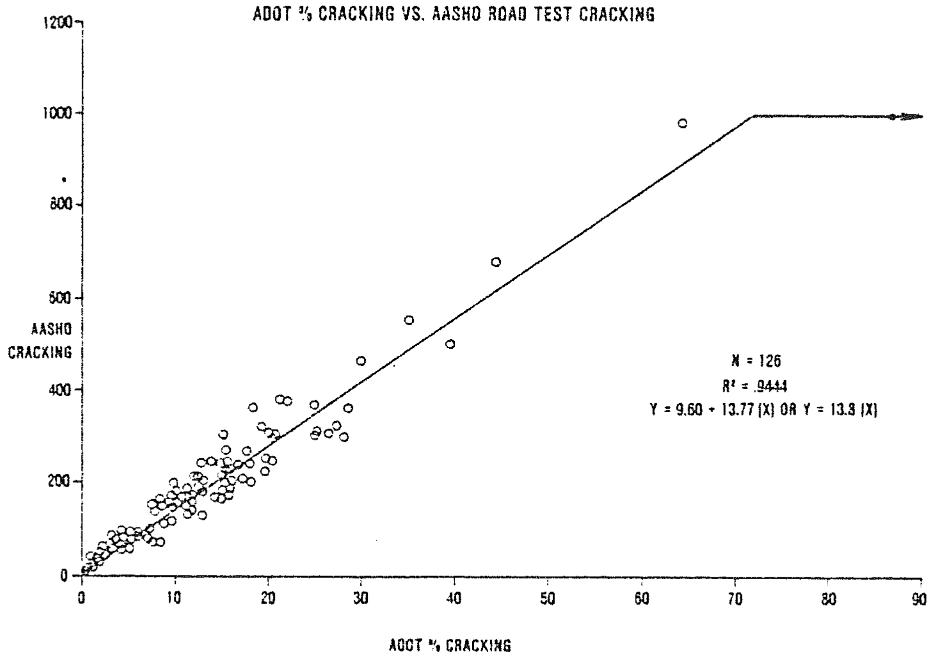


FIGURE 4

ADOT % CRACKING VS. AASHO ROAD TEST CRACKING



Interpretation - New Construction

Two cases of predicted future performance were examined and will be interpreted.

Case 1: Prediction at design stage.

For all miles of highway a predicted expected future roughness or cracking was determined and a correlation between actual and predicted values was performed. By examining Appendix D and E values it was possible to determine if there were and relationships between the correlation, standard error or the slope of the correlation line (B) and time in years. Thus making it possible to establish inferences about the equations ability to predict the future.

Roughness

Figure 5 shows the relationship between the correlation squared and time in years. No trend is observed indicating the equations ability to predict future performance reasonably well over a time period of 32 years.

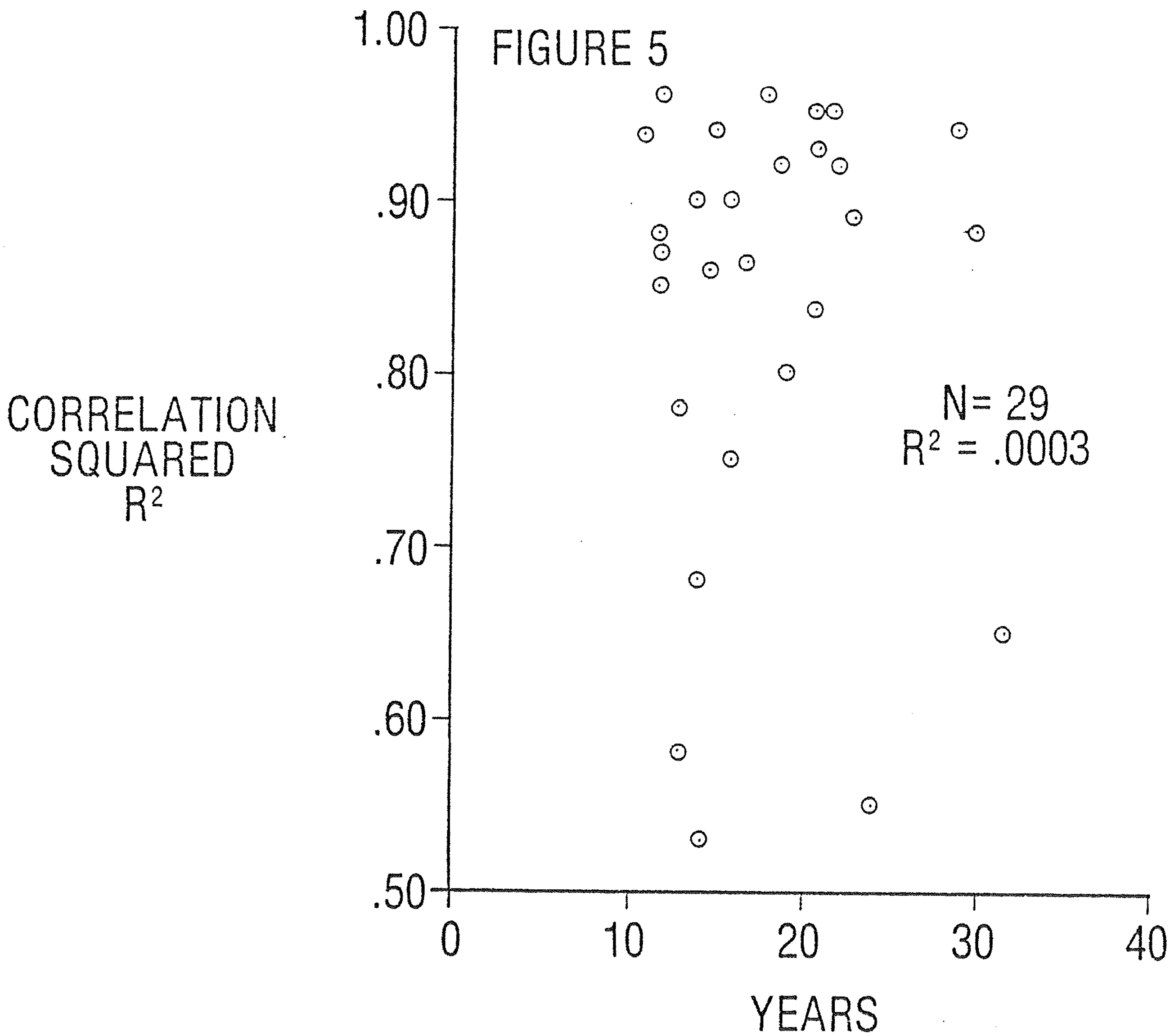


Figure 6 shows the standard error versus time in years. Although the correlation squared value is low, the trend does indicate increasing standard error with time. Hence as the equation predicts into future years the error in prediction increases which is to be expected. Correlations for both Figures 6 and 7 although low are nonetheless good since they represent results expected by project participants.

FIGURE 6

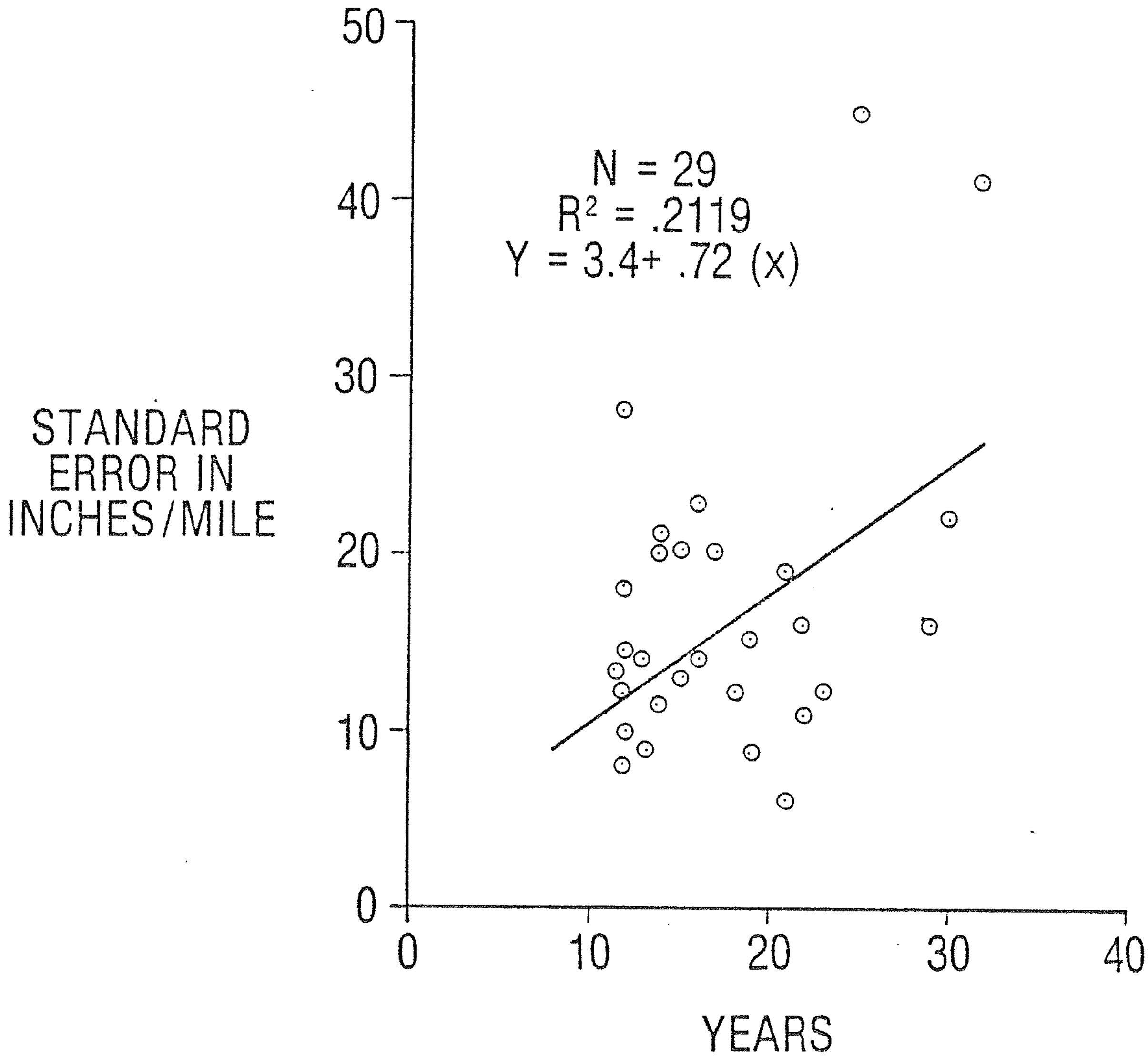


Figure 7 shows the slope of the line (B) versus time. The slope of the line becomes shallower with time. This indicates that as years go by the equation will predict more roughness than actually occurred. It would be most desirable if the slope were close to 1.00. To account for this it is suggested that the average slope of .48 be multiplied times the results from the equation, thus giving an average slope closer to 1.00 and predicted values closer to the acutal magnitude of measured values.

The intercept value (A) was not related to time thus the average value of 19 represents a reasonable correction value, however, this number is small and it is suggested no correction be made to the A coefficient.

Since most highway engineers are very concerned with the life of the pavement the design phase equation gives an opportunity of looking at expected life. Table 1 gives a comparison between the PMS predicted life to a very rough condition and the adjusted PMS predicted life (slope B = .48).

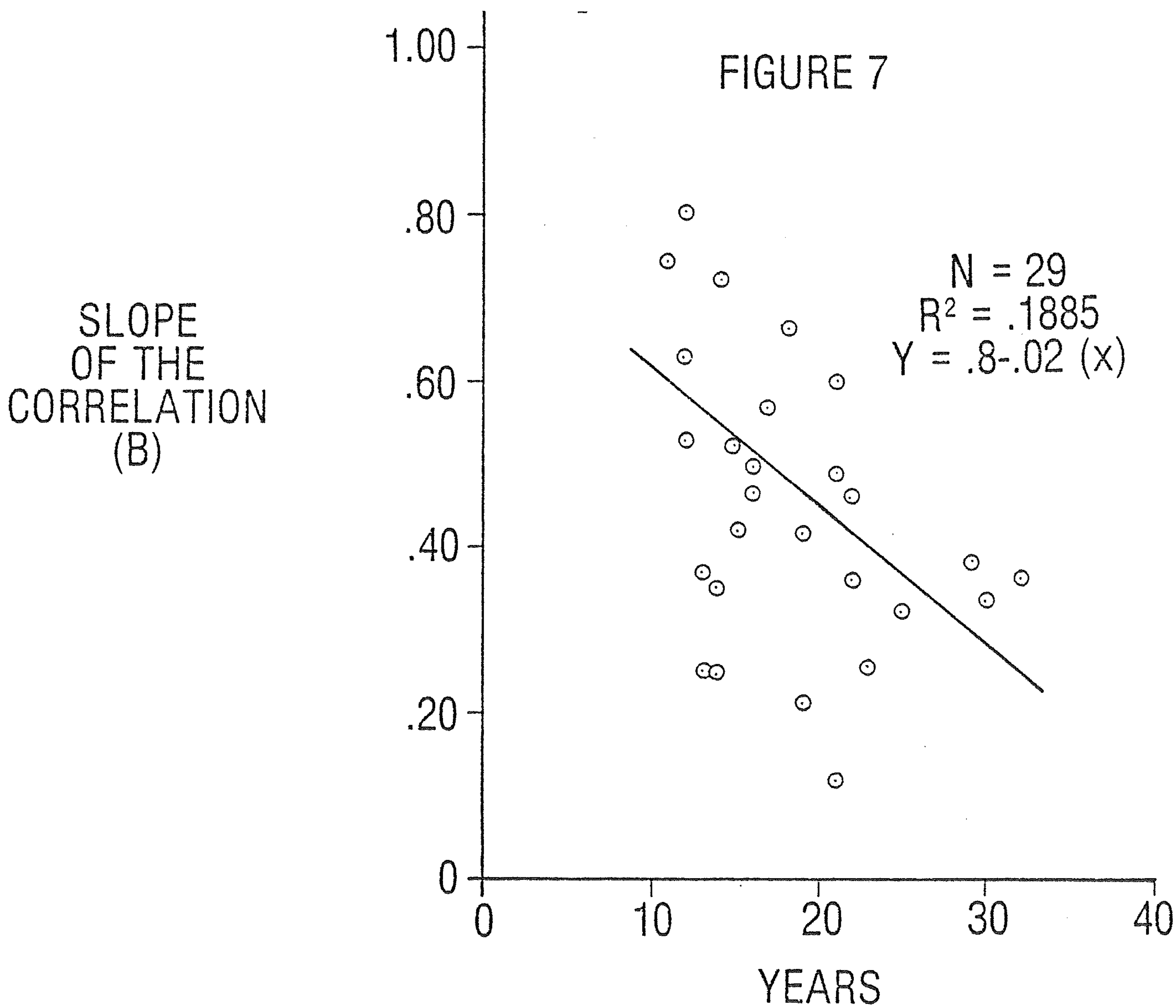


FIGURE 7

Table 1

Years to 256 inches/mile Roughness

	Region	PMS Prediction	Adjusted PMS Prediction
Desert	.5	15	21
	1.0	15	23
	1.5	14	23
Transition	2.0	13	23
	2.5	11	22
Mountains	3.0	9	21
	3.5	7	19
	4.0	6	17
	4.5	5	15

To further substantiate the adjustment sites 12, 14, 19, 21, 24, 28 and 29 all reached the 256 inch/mile value during their life. The average years to this condition was 19 years. The PMS average predicted years to the same condition was 10, whereas the adjusted average predicted years was 21. The adjusted value is much closer to the real world experience.

It was stated earlier in the report that the PMS equations are not design equations, however, they should reasonably well predict future expected distress conditions. In this respect the PMS equations can serve to alert the designer to the expectation that a design will most likely perform in the predicted manner. Given this prediction the designer may choose to reexamine the design to determine if additional structual components (more thickness, stabilization, different asphalt etc.) might be necessary to compensate for future expected distress. Therefore the PMS equations can serve as useful guides to the designer.

Cracking

Figure 8 shows the correlation squared versus time. A strong correlation exists indicating that predictions beyond 20 years should be interpreted as generally poor.

FIGURE 8
% CRACKING

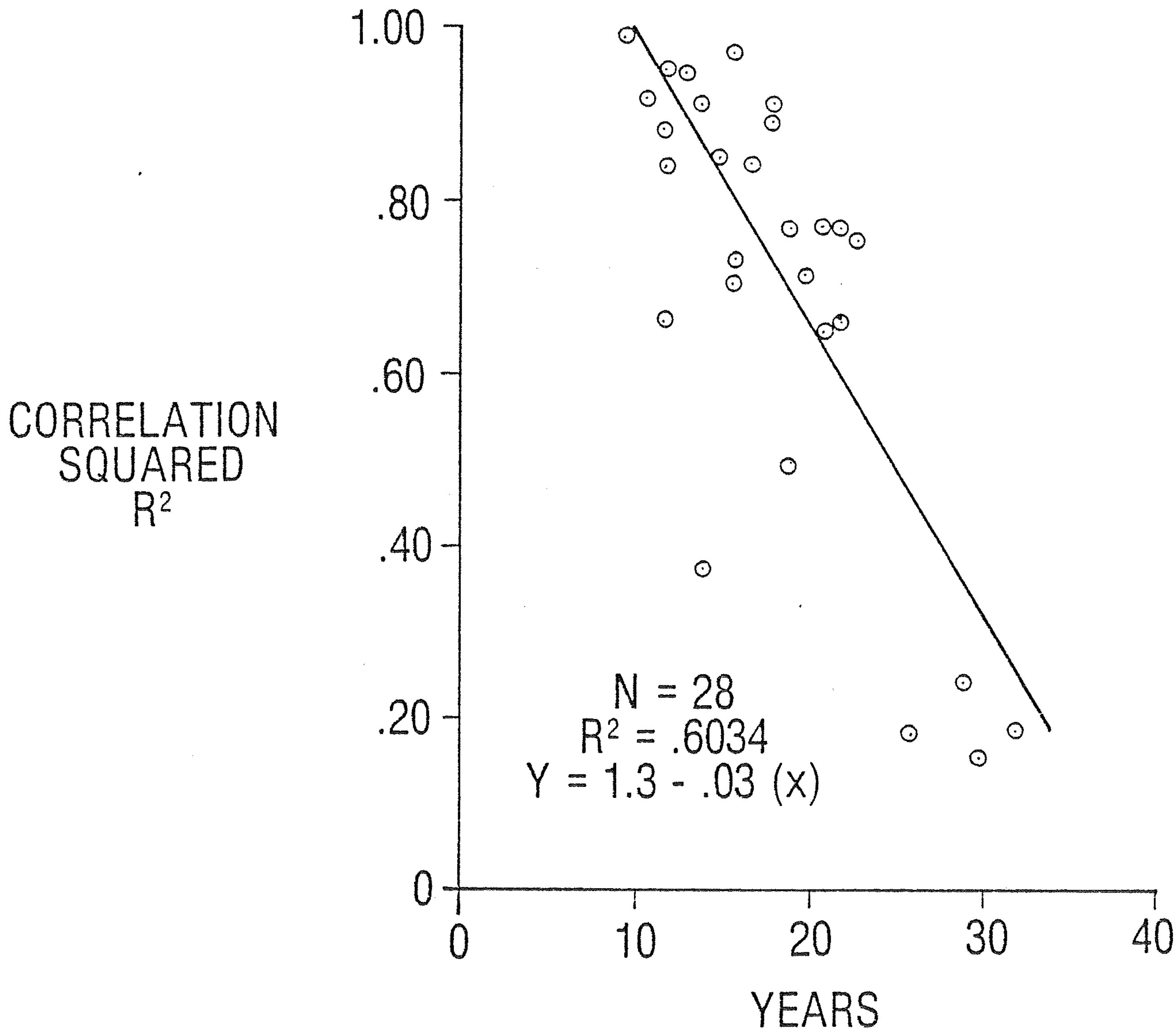


Figure 9 shows that standard error for cracking, like roughness, increases over time. Hence greater error occurs with attempts to predict future cracking.

FIGURE 9
% CRACKING

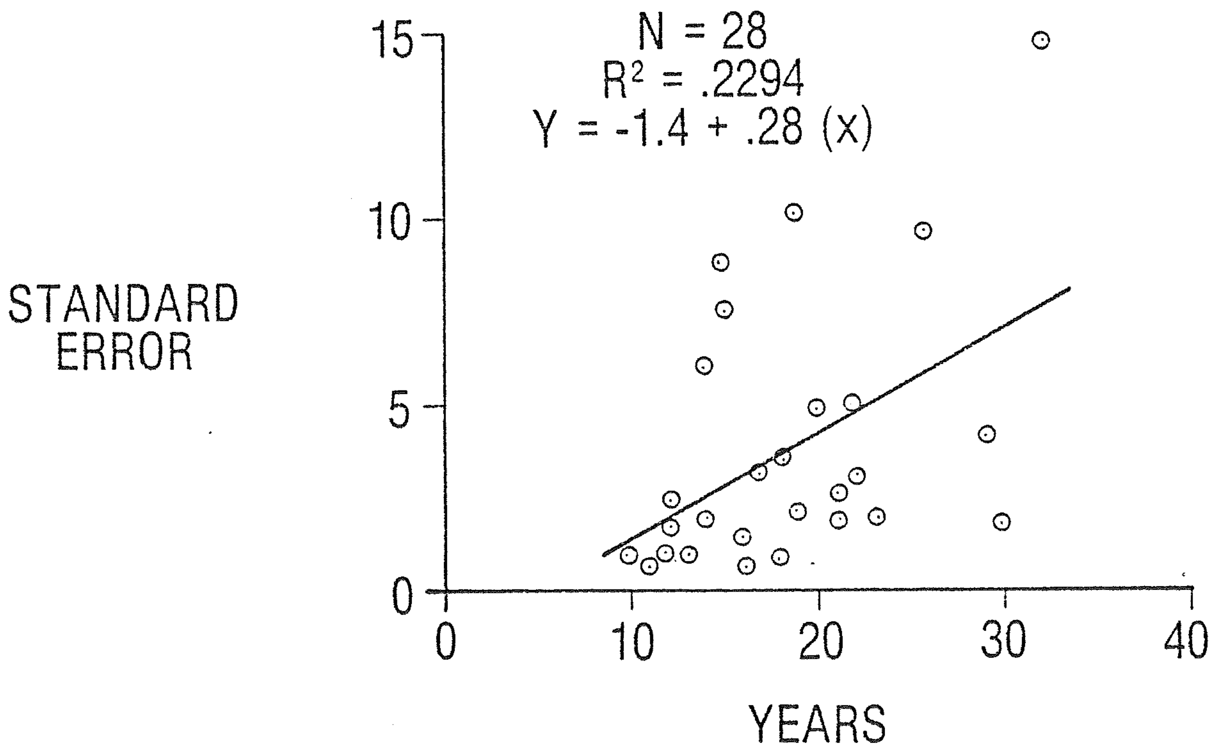
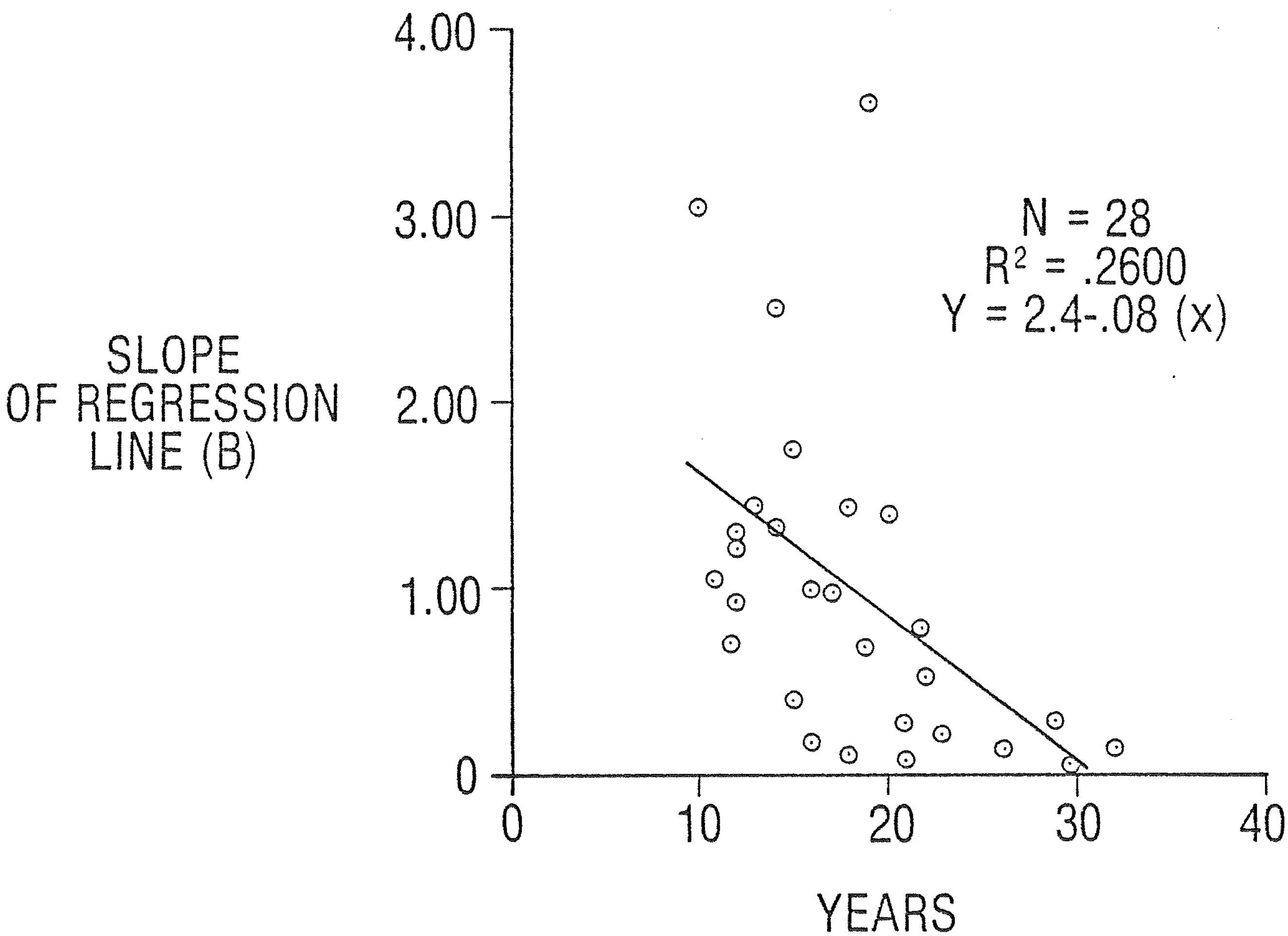


Figure 10 gives the slope (B) versus time. Like roughness slope (B) decreases with time. Over long periods of time greater cracking than occurred will be predicted. Unlike roughness, however, the average slope is close to 1.00, which is of course very disirable. Therefore no correction in slope (B) is suggested. Within the Phase III network optimization system (NOS) only one year predictions are necessary thus no adjustments are needed for NOS.

FIGURE 10
% CRACKING



One of the problems in predicting cracking is the non-continuous nature of cracking. That is the occurrence of the first crack is often delayed many years as shown on Figure II. This figure indicates that pavements built in 1949 first cracked in 1968, 1959 first cracked in 1970 and 1969 first cracked in 1971. There are several possible explanations for this behavior, which include the following:

A). Pavements of 20 years or more of age represent those few remaining structures which exhibited superior performance and would thus indicate exceptionally long crack lives.

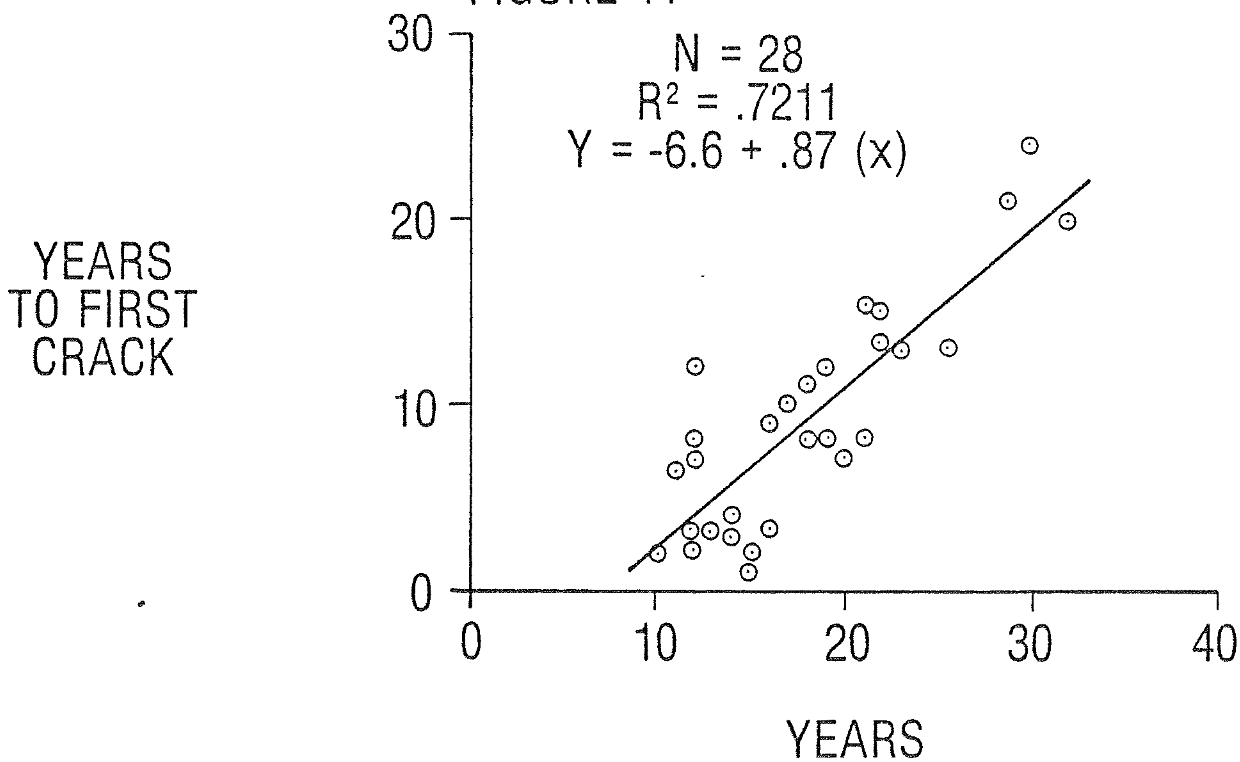
B). Pavements of 20 years or more of age were built to be very flexible, with generally two inches or less of original surfacing. Orginal surfaces generally contained high penetration (200/300) or liquid asphalt (SC-3000).

C). Pavements of 20 years or more of age originally received very light traffic in comparison to todays traffic.

D). Maintenance (seal coats and patches) has tended to cover up cracks, hence masking the true cracking, such that the initial crack survey in 1973 did not see any cracks.

For all the above reasons it is difficult to fairly interpret projects built more than 20 years ago, however, even considering these uncertainties the cracking model in its present form represents a valuable tool to predicting cracking for individual miles of highway up to 20 years.

FIGURE 11



Using the equation it is possible to predict the numbers of years to some future cracking, such as 10 percent. Table 2 shows these values.

Table 2

Years to 10 Percent Cracking

	Region	PMS Prediction
Desert	.5	22
	1.0	16
	1.5	13
Transistion	2.0	12
	2.5	11
Mountains	3.0	10
	3.5	9
	4.0	8
	4.5	8

Comparing those sites which reached 10 percent cracking to the predicted number of years gave the following.

26 sites reached 10 percent cracking

Actual Average Number of Years to 10 Percent	Predicted Average Number of Years to 10 Percent
15	12

Considering all the uncertainties in predicting cracking this is very good agreement. For sites built in the last 20 years the agreement is even better with the actual average number of years being 12 years and predicted 13 years.

In summary the PMS prediction models for both roughness and cracking for case 1, prediction at the design stage, is remarkably good considering the uncertainties in site specific prediction.

Case 2: Prediction given an existing condition in the field.

For all miles of highway a predicted expected future roughness or cracking was determined for each future year based upon an existing condition.

Roughness

Roughness measurements have been taken since 1972, hence only those actually measured values were used in this part of the interpretation. Table 3 summarizes results of this work.

Table 3

Correlation Between Predicted Future Ride in Years 1 Thru 7 Based on a Measured Ride Now. Case 2

Future Year Ride Predicted	N	R^2	Standard Error	A	B	Coefficient of Variation C.V.
1	195	.8922	25.4	9.2	.90	12%
2	169	.8622	28.7	12.6	.84	14
3	139	.8327	31.4	12.9	.80	16
4	111	.8144	33.4	15.8	.75	17
5	82	.8047	34.8	16.0	.73	18
6	53	.8066	34.6	19.7	.70	17
7	25	.8085	36.6	5.9	.74	18

The values in this table clearly show that the PMS equation is very good in predicting the future roughness condition given the present existing pavement condition. The coefficient of variation is below 20 percent from 1 year to 7 years which is also very good, considering the uncertainty of the future. It should be noted that the slope (B) decreases with time. This is similar to the case 1 trend. In order to equate the predicted values more closely to the actual values in terms of magnitude it is suggested that an adjustment factor be used, which is equal to the slope (B) up to four years and then set equal to .70 for five or more years.

In general the PMS equation is capable of predicting future roughness extremely well given the existing condition of the highway.

Preditions of cracking with small standard errors (below 20 percent coefficient of variation) are at best very difficult to make due to large increases in cracking that can and do occur in one year. With this in mind the present PMS equation is considered to be a very good prediction model.

Table 4

Correlation Between Predicted Percent Cracking in Years 1 Thru 6 Based on a Measured Percent Cracking Now

Future Year % Cracking	N	R^2	Standard Error	A	B	Coefficient Of Variation
1	163	.9186	4.0	1.8	.89	12%
2	136	.8266	6.0	4.5	.72	18
3	107	.6435	9.0	8.0	.55	28
4	79	.6158	9.7	10.2	.53	30
5	49	.6068	10.0	12.8	.45	31
6	20	.7091	8.5	13.2	.42	26

Correlation squared (R^2) values, although lower than the roughness values, are still quite good. The standard error and coefficient of variation are above 20 percent an indication of how dramatic increases in cracking can occur in the field. The slope (B) value decreases with time and should be used to adjust the predicted cracking values back down to magnitudes closer to those observed in the field. For those years beyond five or more an adjustment factor of .40 is suggested.

In summary the new PMS equations for both roughness and cracking for both case 1 and 2 can do a very good job of predicting future pavement distress conditions.

This is possible because the models (equations) are of a recursive form. The logic behind a recursive model is that a future condition is dependent upon a past condition. Thus more roughness or cracking accelerater the rate of progression to still more and more roughness and cracking until the pavement has lost its desirable serviceability and structual characteristics. To demonstrate still further how the recursive model emulates the real world additional investigations were performed.

SOMSAC

In the first PMS project Woodward-Clyde developed a model to predict future ride through a Bayesian statistical approach using extensive interviews with knowledgeable highway engineers (1). To further compliment this report a similar set of predicted future roughness values was developed by using the SOMSAC equation in a recursive mode. Results of this comparison can be found on Appendix D and E. In terms of case 1 (Prediction at design stage) the SOMSAC correlations compared on the average as shown in Table 5.

Table 5

PMS - SOMSAC Comparison for Case 1 - Same Sites; Same Time Frame

Average	PMS	SOMSAC
Correlation Squared R^2	.8431	.8111
Standard Error	16.4	18.5
A	18.8	30.4
B	.48	.35
Coefficient of Variation	14%	15%

Even though the PMS model contains fewer terms than SOMSAC, it does a better job of predicting. This is not too surprising because PMS was developed with real observations whereas SOMSAC was developed by guessing. What is surprising is how well SOMSAC predicted considering that no data or observations were used. Evidently highway engineers tended to feel that pavements would last a considerably shorter length of time than actually observed. This can be seen in the two slope (B) values. To understand this three equivalent examples were selected to portray the differences between PMS and SOMSAC predictions in terms of years to an objectionable roughness (256 inches/mile).

Predicted Years to 256 inches/mile Case 1

AASHO REGION	SOMSAC REGION	AJUSTED PMS	PMS	SOMSAC*
1.0	1.0	23	15	11
2.0	2.0	23	13	7
3.0	2.0	21	9	7

* Deflection .001 inch
Traffic 50,000 18 kip per year
Thickness 6 inches of AC
Traffic Growth 1.05

The SOMSAC model predicts an objectionable ride will occur sooner then it actually does by as many as 12 to 14 years.

Case 2 (Prediction given on existing condition in the field) comparisons are shown on table 6. Detailed values by year are shown Appendix D.

Table 6

PMS - SOMSAC Comparison for Case 2 Same Sites; Same Time Frame Years 1 Thru 7

Average	PMS	SOMSAC
Correlation Squared R^2	.8316	.6639
Standard Error	32.1	44.5
A	32.2	28.4
B	.78	.73
Coefficient of Variation	68%	22%

Not surprisingly the PMS equation is again better than the SOMSAC equation. Interestingly though, in this mode the SOMSAC slope (B) is quite close to the PMS value, however, the correlation and scatter are not as good. Examining both Case 1 and 2 the PMS equation is better than the SOMSAC equation.

Present Serviceability Index - PSI

In addition to trying SOMSAC, the PSI design equation for flexible pavements (2) was also tried in the recursive mode. By incrementing traffic it was possible to calculate future expected PSI values and compare them to measured values. Table 7 shows results of this comparison.

Table 7

PMS - PSI Comparison for Case 1

Average	PMS	PSI
Correlation Squared R^2	.8431	.8677
Standard Error	16.4	.7
A	18.8	-14.7
B	.48	4.6
Coefficient of Variation	14%	5%

Using the recursive approach the PSI equation can do a good job of predicting the future PSI, however, the equation is not applicable to all cases. Of the 29 sites it was possible to use the PSI equation on 16, for the other 13, irrational values were calculated. This was primarily due to the low structural numbers (S_N). Such numbers when combined with the 18 kip traffic loading gave ridiculous answers. Therefore the use of the PSI equations for the entire network would be very difficult. In addition using PSI equation routinely would mean the annual collection of considerable more data than is currently collected.

Examining the PSI prediction equations per each site a very large slope (B) value is determined. This equates to predicting much longer lives than actually occurred. To see this table 8 was created. Generally predicted years to 2.5 PSI tend to be more than observed. Another indication can be seen by looking at pavements built since 1963 (year AASHTO interim guides came into use in Arizona). Table 9 shows that 33 percent of the sites have already reached the 2.5 PSI level. This is yet another indication that the PSI equation tends to predict longer lives than are actually observed.

Table 8

AASHTO Region	Actual Years* To 2.5 PSI	Predicted Years to** 2.5 PSI
1.0	18	36
2.0	14	25
3.0	12	20

*Rough average of actual sites

**Traffic 50,000 18 kip per year beginning

SN = 3.88 (6 inch AC, 14 inch base)

SS = 5.00

PSI at beginning = 4.20

Traffic growth = 1.05

Table 9

Number of Sites Built Since 1963 = 15

Sites which Reached 2.5 PSI Since Construction		Sites which did not reach 2.5 PSI Since Construction	
Years of age	No.	Years of age	No.
6	1	8	1
7	1	10	2
14	2	12	5
16	1	14	2
Total	5	Total	10

The PSI equation could also be used in a recursive mode for Case 2 (Prediction given an existing PSI condition in the field). Table 10 shows a comparison between the PMS and PSI for a Case 2 mode. The PSI does not predict as well as the PMS equation in this mode. This is not too surprising since the PSI equation was not developed with this use in mind. In addition the equation is based on AASHTO road test data not Arizona data. Even with these stipulations the recursive mode isn't totally bad.

Table 10

PMS - PSI Comparison for Case 2

Years 1 Thru 7

Average	PMS	PSI
Correlation Squared R^2	.8316	.6959
Standard Error	32.1	.33
A	13.2	.56
B	.78	.76
Coefficient of Variation	16%	10%

In summary two additional approaches to predicting future pavement conditions using a recursive form of the equations were tried. Both approaches give reasonably good approximations given the fact that neither one was specifically designed using Arizona data. In examining both the SOMSAC and PSI equations several trends were observed and adjustments suggested. The PMS equation for roughness appear to be a very useful inventory predictor of future roughness. For purposes of design either SOMSAC of AASHTO should be adjusted to give closer approximations.

Overlays

Both Case 1 and 2 were similarly examined for overlay sites. Appendix A, D and E give detailed data for overlay sites. In all 24 overlay sites were examined.

Roughness

Case 1

Unlike the new construction Case 1 very little correlation was found between years and correlation squared (R^2), standard error or slope (B) as can be seen in Table II. These values are good since they indicate no bias with time.

Table 11

Correlation to Future Years

x=years

Y	N	R^2
R^2	24	.0576
Standard Error	24	.0477
Slope (B)	24	.0332

Hence average of all site values appear to be reasonable indicators. Average values are as follows for the 24 sites.

Average Values

Correlation squared (R^2)	= .7193
Standard error	= 13.04
A intercept	= 20.68
Slope (B)	= .42
Coefficient of Variation	= 18%

Of note again is the low slope (B) value, which as in the new construction work, indicates an overprediction of future roughness. It is suggested an adjustment of .42 be made to the equation thus giving more reasonable answers. The overlay PMS equation is the same as the new construction equation except a second equation adjusts the future predicted roughness based upon the existing highways present roughness. To demonstrate this, plus the adjustment, Table 12 was developed.

Table 12

Overlay* Years to 256 Inches/Mile

Present Roughness Inches/Mile Before Overlay	ASHTO Region	Roughness PMS	Adjusted PMS
250	1.0	15	25
	3.0	9	25
350	1.0	11	20
	3.0	8	20

*2 inch overlay

Since the slope (B) is .42 for overlays and .48 for new construction this would indicate that even thin overlays are capable of maintaining the ride for about the same number of years as the new construction.

Cracking - Case 1

Correlations between years and correlation squared (R^2), standard error or slope (B) shows no trend as the figures below show. As in roughness the average site values can be used as reasonable indicators. Since there is no bias with time.

Correlation to Future Years

Y	N	R^2
R^2	24	.0398
Standard Error	24	.0320
Slope (B)	24	.0162

Average Values

Correlation squared (R^2)	=	.8414
Standard Error	=	.75
A Intercept	=	.78
Slope (B)	=	1.04
Coefficient of Variation	=	22%

The correlation is quite good, standard error low and slope (B) very close to 1.00, which makes this an excellent predictive equation. To enhance the meaning of these numbers table 13 was prepared to show the number of years to 10 percent cracking for various thicknesses of overlay and differing regions. Time to 10 percent cracking is much shorter than the time to 256 inches/mile roughness.

Table 13

Years to 10 Percent Cracking

Overlay	Region	
Thickness	1.0	3.0
1.5 inch	10	9
2.5 inch	12	9
3.5	16	10

ADT = 5000

Years to 10 percent cracking for overlays, compared to new construction (Table 2) show that overlays tend to perform in a manner very similar to the new construction.

In summary both the roughness and percent cracking PMS equations for overlays do a very good job of predicting future conditions.

Roughness

Case 2 - Prediction given an existing condition.

Table 14 summarizes results of the calculations. Although the correlation squared (R^2) is lower than for new construction, the other values would indicate a good correlation.

Since the slope (B) changes with time it is suggested the average slope (B) (.66) be used as an adjustment factor.

Table 14

Correlation Between Predicted Future Ride in Years 1 Thru 7 Based On A Measured Ride Now.

Future Year Ride Predicted	N	R^2	Standard Error	A	B	Coefficient of Variation
1	161	.6555	20.9	16.5	.75	22
2	138	.6107	22.6	16.7	.71	24
3	115	.6607	21.7	8.7	.74	22
4	92	.5777	25.1	11.7	.66	26
5	69	.5944	25.8	10.9	.66	26
6	44	.5952	23.5	25.6	.54	21
7	23	.6760	22.4	11.6	.56	21

Cracking

Case 2 - Prediction given an existing condition.

Table 15 summarizes the various correlation statistics for this care. Although the correlation values fall off by year four, the error terms are not excessively large and the slope (B) value is still good. Predictions for four or more years should be adjusted by using a .75 value to give more reasonable answers.

In summary both the roughness and cracking PMS equations for routine overlays appear to do a good job of predicting the future expected conditions. As an additional reinforcement of the recursive equation mode two additional overlay equations were examined.

Table 15

Correlation Between Predicted Future Percent Cracking In Years 1 Thru 5 Based On A Measured Percent Cracking Now.

Future Year Percent Cracking Predicted	N	R^2	Standard Error	A	B	Coefficient of Variation
1	124	.7520	1.82	.3	.98	15
2	103	.6810	2.14	.4	.96	17
3	79	.5316	2.74	.8	.91	22
4	57	.3587	3.49	1.8	.74	28
5	34	.3514	4.04	1.9	.76	32

SOMSAC

Case 1 and 2

Using the SOMSAC overlay equation for overlay sites in Appendix C, it was possible to do a similar investigation and compare it to the PMS equation. Table 16 shows comparisons for both Case 1 and 2 for PMS and SOMSAC. This table shows that PMS and SOMSAC give surprisingly similar values in terms of correlation, standard error and slope (B). Either PMS or SOMSAC could be used for prediction, however, SOMSAC like PMS would need to have an adjustment factor to account for the differences in slope (B). For Case 1, an adjustment factor of .46 should be multiplied times the SOMSAC value to give reasonable results. To demonstrate this adjustment is shown in the following table.

Table 16

Case 1

PMS - SOMSAC Comparison; Overlays

Average Values	PMS	SOMSAC
Correlation Squared (R^2)	.7193	.7214
Standard Error	13.04	11.32
A Intercept	20.68	22.08
Slope (B)	.42	.46
Coefficient of Variation	18%	15%

Case 2

Average Values	PMS	SOMSAC
Correlation Squared (R^2)	.6243	.5358
Standard Error	23.1	25.8
A Intercept	14.5	.7
Slope (B)	.66	.77
Coefficient of Variation	23%	26%

In summary either PMS or SOMSAC could be used to predict the future roughness of overlays.

Present Serviceability Index (PSI)

Table 17 gives a comparison of PSI to PMS statistics for both Case 1 and 2. Detailed PSI statistics can be found in Appendix D and E.

Years to 256 inches/mile

Overlay, Case 1

SOMSAC Region	SOMSAC*	Adjusted SOMSAC
1.0	15	25
2.0	9	21

*.001 inch deflection
50,000 18 kip single axle EQ./year
2.0 inch AC overlay
50 inches/mile roughness after overlay
1.05 growth in traffic/year

Table 17

Case 1

PMS - PSI Comparison; Overlays

Average Values	PMS	PSI
Correlation Squared (R^2)	.7193	.7750
Standard Error	23.1	.36
A Intercept	14.5	1.82
Slope (B)	.66	.47
Coefficient of Variation	23%	10%

PMS and PSI both do a good job of predicting the case 1 future condition, however, PMS is much better than PSI for case 2. The good showing is additional testimony to the premise that a recursive form of a pavement prediction equation is a reasonable model of what really occurs in the field. Interestingly for case 2 PSI has a slope (B) of 5.01 and for case 2 a slope (B) of .47. This is very similar to the new construction case 1 and 2 results shown on Table 7 and 10. The 5.01 value would indicate that the PSI equation for overlays predicts more years of service than actually occurs by values similar to Table 8.

By examining both SOMSAC and PSI equations in a recursive mode it has been demonstrated that the PMS equation can give comparably good predictions of future performance. All three equations need some adjustment for either case 1 or 2 or both in order to more closely approximate actual performance. This section has dealt with conventional overlays, however, overlays with special treatments (asphalt rubber, heater scarification) have also been built and will be examined.

Special Treatments with Overlays

Over the years ADOT has used either heater scarification or asphalt rubber to improve roughness and cracking performance of overlays. Generally such treatments have been employed when unusual amounts of cracking (greater than 10 percent) have been present in the existing road. In addition they have been employed when no other conventional material or process short of reconstruction appeared capable of providing satisfactory performance. Therefore when either conventional overlays or special treatment performance is observed it should be recalled that generally both heater scarification and asphalt rubber were used where the degree of difficulty in improving performance was indeed much higher than a routine conventional overlay. It should also be mentioned that extensive use of special treatments as part of the routine overlay design strategies is relatively new, which means the data base of field performance is limited. Numerous special research reports have been issued documenting performance (4) (5) (6) (7). Indeed reference (7) reports on the performance of all asphalt rubber projects. A similar report will be forthcoming next year or all heater scarification projects. With these thoughts in mind nine miles of heater scarification and nine miles of asphalt rubber were selected from different projects and are listed on Appendix F.

Results of this analysis are grouped by treatment and case.

Asphalt Rubber

Case 1 and 2 - Ride and Cracking

Both the ride and cracking statistics for case 1 and 2 are shown on table 17. The ride values are not too good primarily due to the limited nature of the data. Only five years of data have been collected up until now. The range of ride values is very limted. The standard error and coefficient of variation values are reasonable and are indication that the model is performing as intended. Slope (B) values are smaller than one indicating a longer than expected life, however, current expected lives already are predicted to be 20 years. Given that the current performance trend represents only five years of actual data it is felt that adjustments at this time would be unwise. The cracking predictions for the five year period is remarkably good. The cracking equation predicted no cracking and up until now there has been no cracking.

Table 18

Asphalt Rubber Case 1

Average	Ride	Cracking
Correlation Squared (R^2)	.5777	1.0000
Standard Error	12.6	0.0
A Intercept	44.3	0.0
B Slope	.70	1.00
Coefficient of Variation	17	0.0

Asphalt Rubber Case 2

Average	Ride	Cracking
Correlation Squared (R^2)	.3238	1.0000
Standard Error	31.3	0.0
A Intercept	39.0	0.0
B Slope	.53	1.00
Coefficient of Variation	33	0

Heater Scarification

Case 1 and 2 - Ride and Cracking

Statistics for both cases are shown on Table 18. As in the cracking case the ride values are not too good, however, a maximum of only 9 years of ride history is known. In addition virtually all the ride values are still in the good range, thus restricting the size of numbers considerably. At present the PMS equation seems capable of giving good ride correlation in the future. Cracking statistics are very good for both cases indicating that the PMS cracking equation has good prediction capabilities.

Table 19

Heater Scarification Case 1

Average	Ride	Cracking
Correlation Squared (R^2)	.6239	.8993
Standard Error	13.6	.4
A Intercept	-7.2	.1
B Slope	1.23	.95
Coefficient of Variation	17	18

Heater Scarification Case 2

Average	Ride	Cracking
Correlation Squared (R^2)	.4489	.9257
Standard Error	22.3	1.2
A Intercept	35.6	-.7
B Slope	.57	1.1
Coefficient of Variation	23	16

In summary the special treatments portion of the PMS overlay equations appears to be a reasonably good approximation of the future performance of these materials. As additional ride and cracking data is collected in future years the equations can be updated and certainly improved.

Concrete - PCCP

Although ADOT has only about 250 miles of concrete highways in its system, it was agreed some prediction model was needed. Historically the ride of concrete pavements has been of major concern, thus a prediction equation using the same approach as the new design and existing flexible pavement equation was utilized. A small sample set of 12 miles of concrete highway was used to generate the predictive equation. The derived equation had poor correlation, however, it was thought that even a poor equation was better than no equation.

PCCP - Ride Equation

$$R_N = 14.73 + .04(R) = 3.00(R_g)$$

R_N = Change in roughness during next year
R = Present roughness
R_g = Regional factor

Correlation Squared (R^2) = .0258

Appendix G gives the raw data used to develop this equation. In addition the raw data and correlations for six other miles of highway are shown. These six additional miles of highway were used to verify the degree of agreement. Table 19 gives the statistical measurements for both case 1 and 2 for the ride prediction. Results show a good correlation with small coefficients of variation. Slope (B) values should be slightly adjusted to .58 for case 1 and .78 for case 2. Considering the above adjustments a comparison of AC to PCCP can be made by using Table 1. Table 19 indicates that plain jointed PCCP (9" slabs) would reach the 256 inch/mile roughness (very rough pavement) in about 60 percent of the time that it would take an AC pavement (or about eight years sooner).

Table 20

Concrete Highways; Jointed PCCP Ride

Average	Case 1	Case 2
Correlation Squared (R^2)	.9028	.7905
Standard Error	21.4	35.9
A Intercept	12.5	10.8
B Slope	.58	.78
Coefficient of Variation	15	16

The PCCP PMS equation appears to be a reasonably good predictor of future performance for both case 1 and 2. This concludes the mathematical verification interpretation.

Table 21

Years to 256 Inches/Mile Roughness AC and PCCP; Case 1

	Region	AC Adjusted PMS	PCCP Adjusted PMS
Desert	1.0	23	15
Transistion	2.0	23	14
Mountains	3.0	21	13

Conclusions

It has been demonstrated that the PMS models (equations) can reasonably predict both the future ride and cracking for AC pavements (new, existing and overlays) and PCCP pavements. Many suggested minor adjustments should be made to produce a reasonable set of models. It should be recalled that this is a start, no doubt future verification calculations will make additional adjustments which will improve the models ability to predict the future.

It appears that both new AC pavements and overlays are capable of providing a comfortable ride up to and beyond 20 years. Generally cracking will start and progress to objectional values in about 10 years unless some special treatment is used which can extend the period of low cracking beyond 10 years.

Concrete highways built out of plain jointed concrete of no more than 9 inches thickness generally reach a rough condition in about 15 years or about 60 percent of the time that AC pavements reach the same condition. Additional work on characterizing the performance of ground PCCP and overlayed PCCP needs to be done in the future.

In terms of Present Serviceability Index (PSI) for AC pavements objectionable levels of service (below 2.5) is reached in less than 20 years. This appears to be due in part to the overprediction of performance which should be further investigated.

The SOMSAC equations are capable of producing reasonably good predictions of future performance. These equations contain terms for deflection and traffic and could be used to check the design of new highways and overlays.

Recommendations

The new PMS prediction models with adjustments should become past of the PMS network optimization program.

The SOMSAC ride equations with adjustments and the PMS, overlay cracking equations contain terms which make them useful as equations to check the designs of both new and overlayed pavements.

Such equations should be incorporated into the SOMSAC program.

A similar verification process should be repeated about once every four years for purposes of testing the equations and evaluating new designs or construction techniques; such of recycling, sulfer asphalt, overlays with special treatment, grinding of concrete and overlaying of concrete.

Additional special investigations which would determine why some miles of highway have not performed as expected are also encouraged.

In closing ADOT has available to it a valuable prediction tool not available in any other state at this time. This valuable tool should be implemented and used as much as possible within the context of management, design and research of pavements within Arizona.

REFERENCES

1. Finn, F.N., Kulkarni, R., McMorran J., "Development of Framework for Pavement Management System for Arizona," Arizona Dept. of Transportation Report ADOT-RS-14(161)-1, December 1976.

2. "AASHTO Interim Guide for Design of Pavement Structures 1972," 1974, American Association of State Highway and Transportation Officials.

3. "Asphalt Overlays and Pavement Rehabilitation," The Asphalt Institute, MS-17, November 1977.

4. Way, George, "Prevention of Reflective Cracking Minnetonka-East (1979 Addendum Report)", Arizona Dept. of Transportation Report 1979-GWI, August 1979.

5. Way, George, "Tests on Treatments for Reflective Cracking," Transportation Research Record 647.

6. Forstie, D., Walsh, H., Way, G., "Membrane Technique for Control of Expansive Clays," Transportation Research Board, January 1979.

7. Gonsalves, G.F.D., "Evaluation of Road Surfaces Utilizing Asphalt Rubber 1978," Arizona Dept. Of Transportation Report Number 1979-663, November 1979.